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SEVENTY-FIVE YEARS OF AGGREGATE RESEARCH IN KANSAS

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16 Abstract

The Kansas Department of Transportation (KDOT) has a long history of aggregate research directed towards finding the most reliable and durable aggregate for highway construction. Beginning with a study on freeze thaw durability in 1928, this paper summarizes the historical development of aggregate research conducted over the last 75 years. Research studies have focused predominantly on freeze-thaw damage (D-cracking) and alkali-silica reaction (ASR). This research has contributed significantly towards the development of current specifications. Today KDOT pavements are relatively free of ASR and D-cracking. Current test methods and concrete aggregate specifications have been included in the Appendix.

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by

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INTRODUCTION

The durability of concrete along with its versatility and economy has made it the world's most widely used construction material. Properly constituted, placed and cured concrete has a long service life under most natural and industrial environments, although premature failures of concrete structures do occur. The deterioration of concrete can be a very complicated phenomenon involving many physical and chemical factors. In colder regions, the most harmful physical factor is often the freeze-thaw cycles whereas alkali-silica reactivity (ASR), a chemical factor, is reported to be a problem for almost all regions. The durability of concrete against these two factors is dictated largely by the durability of its aggregates in such situations. Although concrete construction has been in practice for centuries, it was only during the beginning of this century when researchers started to study and understand the affect of aggregates on the behavior of concrete.

In Kansas, the lack of durability has long been a concern due to the large number of freeze-thaw cycles annually and because of the nature of our local aggregates. The first large-scale investigation on freeze-thaw damage of concrete in Kansas was performed in 1921. The first investigation into the occurrence of map cracking due to alkali-silica reaction was performed in the early 1930s. In 1928, Scholer published a paper [1] on his study of deterioration in concrete structures constructed of Kansas aggregates. Since that time the Kansas Department of Transportation (KDOT) along with different individuals and agencies, has continued to research, study and improve the quality of concrete incorporating Kansas aggregates. Research studies have focused predominantly on freeze-thaw damage and alkali-silica reactivity of aggregates and have significantly contributed to the development of KDOT's current tests and specifications for aggregate durability.

OBJECTIVE

The major objective of this paper is to summarize the historical development of aggregate durability research in the state of Kansas. The specific objectives are:

1. To compile the specifications currently used by KDOT to prevent ASR and freeze-thaw durability problems.
2. To summarize KDOT tests to identify ASR and freeze-thaw damage potential of aggregates.

HISTORY OF FREEZE-THAW DURABILITY RESEARCH IN KANSAS

D-cracking has been a recognized mechanism for over 60 years. It is a form of deterioration found in Portland Cement Concrete Pavement (PCCP) encountered throughout the United States and Canada. D-cracking is associated primarily with the use of certain coarse aggregates in the concrete that disintegrate when they become saturated with water and are subjected to repeated cycles of freezing and thawing. It is identified by the disintegration of concrete characterized by a series of fine, closely spaced cracks parallel to joints and cracks that appear at the wearing surface. These cracks are often filled with a white, blue, gray or black deposit consisting of calcium carbonate and road grime.

Since D-cracking was first identified in Kansas, there have been several studies on the freeze-thaw durability of concrete. The Kansas Department of Transportation initiated studies in 1921, 1944, 1951, 1964, 1967 and 1980 concerning this type of pavement deterioration. Through these studies and research by other agencies, there have been many changes to our specifications over the years. Today, Kansas highways built under the current specifications are essentially free of any distress due to D-cracking.

1921-1941 Study

Laboratory test of soundness of aggregates by means of sodium sulfate or magnesium sulfate had limited success correlating with field service records. The chemical tests correlated well with soft aggregates containing shales, but did not prove satisfactory on some types of cherts and flints. A study [2] of soundness by freezing and thawing of bare aggregate indicated that this test method was a better indicator of field performance. An existing freezing and thawing test method [1] was modified so that the results were based on cumulative percentages retained on a series of sieves. This made it possible to correctly evaluate aggregate pieces which were cracked in crushing and separated once during a cycle of freezing and thawing and did not show real signs of disintegration.

The result of the test was expressed as the ratio of the sum of the cumulative percentages retained on each of the sieves after freezing and thawing to the sum of the cumulative percentages before the test. Analysis of aggregates under both the freeze-thaw and chemical test methods continued for several years. By the year 1931, this test method [3] was the basis of acceptance for soundness by KDOT and is still part of the aggregate acceptance specifications.

1944-1945 Study

In the early 1940s serious deterioration and severe damage was present in many Kansas pavements [4, 5]. The first large-scale investigation [6, 7] into pavement deterioration began in 1944. A field survey [8] of 330 highway projects were examined comprising 1,883 km (1,170 miles) of concrete pavement constructed between 1919-1945. This was approximately 85% of all existing concrete pavements in Kansas. The study concentrated on the relationship between pavement failures and the materials incorporated in the pavement.

Coarse aggregates consisted of five general types: crushed limestone, chert gravels, crushed flint, flint chats, and sand gravel. The sand gravels came from two general sources, the Kansas River valley and the Arkansas River valley, and the cements came from 12 sources.

As the data was analyzed, it was found that the fine aggregate had a relatively small influence with respect to the formation of D-cracking. The same was true of the various Portland cements. However, it was found that the relationship between the coarse aggregate and D-crack failures was quite significant. Some of the coarse aggregates showed very good service records while others showed predominately poor service indicating that it was the limestone aggregates that were associated with D-cracking.

Several changes to the coarse aggregate for concrete specification were made based on the results of this research. Most notably, the aggregate freezing and thawing test procedure and a reduction of the maximum aggregate size were implemented.

1951 Study

The 1951-study [9] was similar to the one conducted in 1944. D-cracking was still appearing in some pavements, but in a very haphazard pattern. No common denominator was readily apparent. A field survey was taken of 1854 km (1,165 miles) of bare concrete constructed between 1921-1945 and 399 km (248 miles) of covered concrete pavement constructed between 1919-1937. The purpose of the study was to determine if materials or design influenced the existence or lack of pavement deterioration. It was found that the older pavements had a poorer rating than the newer pavements, except for newer pavements with a low cement factor 223kg/cm3 (376 lb./yd3) also had poor ratings. Results also indicated that some particular coarse aggregates were more susceptible to deterioration than others, but because of the many variables involved the researchers were not able to pinpoint the cause of the deterioration.

1964 Study

For several years after the 1951 survey, D-cracking continued to be an irresoluble problem. In the mid-1950s Kansas, as well as the rest of the country, saw a marked increased in pavement construction. By 1962 stain patterns began to appear in some new pavements (less than five years old) at joints and uncontrolled cracks. This stained area often progresses into D-cracking and by 1963 D-cracking was appearing in many of these newer pavements. In 1964, another study was initiated with several objectives:

First, to determine the extent of D-cracking damage in Kansas pavements, second, to determine the reason for its occurrence, third, to determine the most effective method of repair and finally, establish any changes in procedures and/or materials needed to prevent D-cracking in new pavements. The result was a series of reports generated over a number of years. The four volumes include the Field Survey, Laboratory Phase, Air Photo Survey, and the Petrographic Survey. The first phase of the study [10], involved a field study to determine the extent of damage. This time 1931 km (1,200 miles) of concrete pavement placed between 1944 and 1965 were evaluated. Conclusions reached were:

1. D-cracking was still a problem in Kansas.
2. All Kansas limestone used in concrete pavements have been associated with D-cracking to various degrees[1].
3. Pavements with limestone coarse aggregate in excess of 35% were more likely to be D-cracked than pavements with less than 35%.
4. Most of the pavements without limestone coarse aggregates were rated good except for a few pavements constructed with some sand-gravels.

Phase two of the study [11] selected 30 locations from the pavements surveyed in phase one of this study. Cores were taken from each site and tested for compressive and tensile strength, water content, density, and freeze-thaw durability. Results concluded that:

1. D-cracking deterioration did not relate to concrete density.
2. Tensile or compressive strengths of pavement cores and air-entrained concretes were the most durable in laboratory freeze-thaw tests.
3. Cores showed high water penetration.
4. Traffic promotes raveling of D-crack surfaces, but not its occurrence.

From aggregate samples representative of the pavements tested, they found that:

1. All of the Kansas aggregates showed low durability factors.
2. Reduced aggregate size did not relate to better freeze-thaw durability.
3. Heat drying of the aggregate prior to use was not beneficial.

Phase three of the study [12] focused on investigating an accelerated method of mapping pavement deterioration with aerial photography. The researchers found that measurement of D-crack related pavement stain through interpretation of low level aerial photography could be used as a maintenance indicator to reveal pavements that might require maintenance. However, an automated system of data analysis was needed because of the large amounts of data acquired.

They also found that interpretation of stain growth might not be feasible when studded tires are allowed on the pavements.

Phase four [13], the final report, was a petrographic analysis of the aggregates and the air void system using the cores taken in phase two of the study. Microscopic examinations found that not all D-cracking began at the bottom of the pavement as once believed and that the limestone aggregates were the locus of the initiating cracks. They also found that many of the coarse aggregates retained moisture longer than the surrounding paste. In addition, it was observed that many individual samples contained limestones of differing textures and colors and within these samples some of the limestone aggregates were more resistant to deterioration than others.

Another part of the petrographic examination involved an analysis of the air void system. The benefits of a proper air void system are well documented and verification was needed to see if the air void system could be part of the problem. The samples examined contained tolerable to good air-void parameters. It was concluded that while good air-void systems did not prevent D-cracking, it was noted that good systems do contribute to longer life in severe climates.

X-ray diffraction was used to determine the possibility of alkali carbonate reactivity. Results indicated that several Kansas limestone aggregates were susceptible to alkali carbonate reaction. Insoluble residue of carbonate aggregates was also investigated as a possible indicator of D-cracking susceptibility. This study determined that the insoluble residue was not of obvious significance with respect to D-cracking, later studies (1980) did find a connection.

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[1] Note that at this time limestone aggregates were not accepted on a ledge by ledge basis as they are today. Sources generally included the full face of the quarry wall, which could include several ledges.

Also included in the study was the possibility of the subbase contributing to the cause of D-cracking. Analysis was based on the gradation, pavement condition, salt content and pH of the subbase. The testing did not indicate that these parameters were significant in the performance of the pavement as related to D-cracking. It should be noted that this report was not published until 1995. In addition to the information discussed above, the report contains observations on aggregate durability as KDOT's knowledge expanded over the years.

1967 Study

A 1967 companion study [14] was conducted by Kansas State University that involved both field and laboratory studies. The objectives were to identify and stop D-cracking in existing pavements as well as reproduce D-cracking in the laboratory. The field study program comprised testing a variety of surface sealers, subsurface grouting, and installation of vertical drains. The surface treatments included linseed oil, commercial silicone, and commercial resin. The subsurface grouting was comprised of two commercial silicate based grouts and linseed oil emulsion. All of these treatments failed to slow or stop D-cracking. Wick type vertical drains tested for removing base course moisture were also ineffective. Laboratory results indicated that no free lime was found in Kansas limestones and that bubbling pressure techniques could be used to determine the pore size in limestone aggregate, but did not indicate D-cracking susceptibility. D-cracking was recreated in the lab, but the mechanism was still not understood.

1980 Study

The 1980 study [15, 16] was instituted in response to a FHWA mandate. In essence, the mandate required that KDOT make some significant progress toward the elimination of D-cracking if Federal funds were to be approved for PCCP. Since most of the concrete pavement being constructed was federally funded it would have effectively eliminated PCCP as a surfacing option in most cases.

<u>Literature Review and Proposed Tests</u>

A literature review was conducted for insight into various testing methods. Due to time and monetary considerations the initial test procedures chosen were:

- Aggregate Soundness @ 25 cycles of Freeze and Thaw (Freeze-thaw Soundness)
- 24 Hour Water Absorption
- Acid Insoluble Residue
- Iowa Pore Index

It was also decided that the ultimate approval would be based on ASTM C 666 Procedure B results. This was based on several factors, mainly C 666 was becoming the test of choice for several agencies at this time and KDOT's earlier freeze-thaw studies indicated that ultimately this type of test would provide the best field correlation.

A field survey was undertaken to assess the 1980 condition of pavements constructed between 1961 and 1974. Test results were available for many of the coarse aggregates in these pavements and the time range covered all ages of pavements up to the 20-year design period. Pavements less than five years were considered too recent to be of significant value to the study. The pavements were rated on a scale of zero, no evidence of deterioration, to ten for severe deterioration. Approximately 279 miles under 76 separate projects were rated in this phase. Each section was photographed with the location referenced to mileposts and cores were taken from pavements showing advanced distress. These results were used to judge how well the various laboratory tests could predict field performance.

Quarry Site Investigation

All limestone quarries producing aggregates for use on Kansas Department of Transportation projects were sampled and detailed by KDOT geologists. Sampling consisted of taking ledge samples from each separate bed for testing. Beds in this sense are visually discernable layers within the quarry face separated by seams, partings or lithologic differences. Detailing consisted of determining the geologic classification, placement, thickness and lithologic description of each bed. A permanent photographic record was also taken in order to allow field re-establishment of the original beds later. The photo on the following page is an example of a quarry face divided into ledges.

Testing Program

Initial testing was performed in an attempt to correlate the Iowa Pore Index with other properties determined by routine testing of quality and assurance samples. General trends noted were:

1. For a given aggregate freeze-thaw soundness, higher absorption indicated lower pore indices.
2. Higher amounts of acid insoluble residue indicated higher pore indices.

However, the pore indices determined did not agree with observed field conditions to a satisfactory degree.

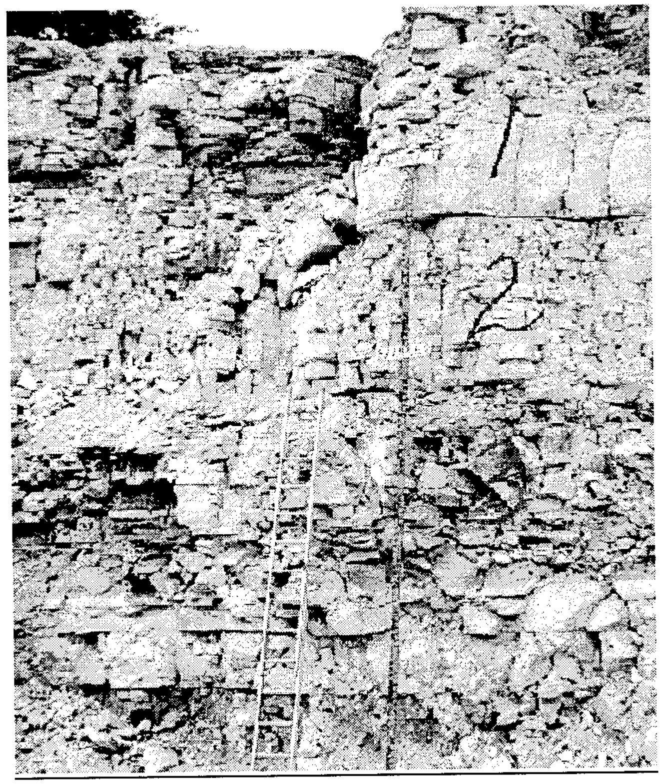

Figure 1. Typical Quarry Face Indicating Lithologic Ledges

As ASTM C 666 (Procedure B) results became available, it was found that further inconsistencies were noted between those results and the Iowa Pore Index results. The Iowa Pore Index failed to discriminate between good and poor aggregate performance.

The Aggregate Soundness Ratio also failed to predict C 666 durability or expansion. Single parameters such as Freeze-Thaw Soundness, 24-Hour Water Absorption, and Acid Insoluble Residue Tests all failed to give a discernable pattern for prediction for freeze-thaw durability.

There was, however, a general trend of lesser durability for aggregates with higher percentages of acid insoluble residue; and for a given value of (unconfined) aggregate Freeze Thaw Soundness, higher absorption generally accompanied higher durability. This trend indicated that for a given value of Acid Insoluble Residue a minimum value of the 24-Hour Water Absorption would be appropriate to obtain a high durability factor. Conversely, for a given water absorption, a maximum acid insoluble residue content would be appropriate. Resolution of this observation into a single factor was accomplished by the Pavement Vulnerability Factor (PVF).

In a paper by Bisque [17], it was proposed that during the formation of carbonate rocks and in subsequent recrystallization, a concentration of the siliceous constituents would occur at the grain boundaries (the pore wall of the rock). Subsequent secondary silicification would also result in the formation of siliceous materials within the pore spaces. For the sake of simplicity, four assumptions were made:

(1) Only minor amounts of clay were present.

(2) The acid insoluble residue was of constant specific gravity (2.60 for quartz).

(3) The 24-hour absorption would approximate the remaining porosity volume.

(4) The initial porosity could be presumed to be the total acid insoluble residue volume plus the remaining porosity volume.

By doing so, it allowed the volumetric loss of initial porosity (due to acid insoluble residue) to be computed on a percentage basis. This percentage value was given the name Pavement Vulnerability Factor (PVF). It is computed from the following equation:

$$PFV = \frac{100\ A}{A + \frac{B}{0.3846}}$$

where: A = Percentage by weight of Acid Insoluble Residue

B = Percentage by weight of Water Absorption

$$0.3846 = \frac{1}{2.60}$$

Testing Results

Results of the various tests conducted were correlated to the results of ASTM C 666, Procedure B for each ledge sample collected. Details are as follows:

- Iowa Pore Index - As previously stated, the Iowa Pore Index failed to give a sufficient prediction criteria for good or poor performance in concrete prisms tested under ASTM C 666 (B) for Kansas limestones.
- 24-Hour Water Absorption - It was found that the effect of absorption upon concrete prism durability is negligible. In general, the range of absorption values for durable aggregate is no different from the range for non-durable aggregates.

- Acid Insoluble Residue - As previously noted, a general trend exists in which higher acid-insoluble contents accompany the concrete prism with lower durability.
- PVF – The main group of low durability aggregates could be eliminated by setting an appropriate maximum PVF.

Various combinations of the results were examined. It was found that fixing the PVF limit at 40 (maximum) and freeze-thaw soundness ratio at 0.95 (minimum) would virtually eliminate all aggregate samples with a low durability factor. However, the statistical treatment of the data indicated that some risk of obtaining a material with a low Durability Factor existed. Using this perspective, the addition of Acid Insoluble Residue was added to the combination with a maximum limit of 3.50% which eliminated all known aggregates with a durability factor below 95 (the accepted lower limit).

This combination was used to create Class VI aggregate [18] for concrete. It allowed for the relatively quick acceptance of an aggregate "bed" pending results from ASTM C 666 (B).

Implementation

Based on the finding of this study, a Special Provision [19] to the Standard Specifications was initiated in April 1981. It provided for two acceptable classes of limestone aggregate for concrete pavements, these are listed in Table 1.

Table 1. Durability Specifications for 19mm Maximum Size Limestone Aggregates [9].

Class	Durability Factor (min.)*	Expansion (%) (maximum)*	Soundness (Mod. Freeze-thaw) (minimum)	PVF (max.)	Acid Insoluble Residue (%) (maximum)
I	95	0.025	0.85		
VI			0.95	35	3.5

* from ASTM C 666; Procedure B (sample curing condition modified to 300 cycles)

Conclusions

The study resulted in the following conclusions and observations:

1. Aggregate test methods and specifications in effect in Kansas prior to the inception of the Durability Class ratings failed to assure satisfactory performance.
2. Separation into discernable beds for purposes of pre-qualification is necessary in most instances to assure satisfactory materials.
3. Modifying ASTM C 666, Procedure B to include 90 days of cure before the cycles of freezing and thawing began produced better correlation with field service records.
4. The use of the PVF factor has shown a satisfactory relationship between laboratory test and field performance.
5. The use of the PVF factor is a reliable indicator of potential durability for those limestones examined to date.
6. The use of the PVF factor for limestones dissimilar to those examined requires additional study.

1986 Evaluation

In 1986, KDOT initiated an experimental test section on US 169 in Johnson County to evaluate the effectiveness of Special Provision 80P–103. Four sections of pavement, approximately 457 m (1,500 ft.) long, were constructed each representing different levels of expected durability. The test sections have been evaluated on a yearly basis.

Section 1 was representative of a Class I aggregate. Section 2 was chosen as a test of the sensitivity of the Durability Factor. Allowable limits were 95 for the durability factor and 0.025% for expansion. The coarse aggregate supplied barely met the requirements of these limits. Section 3 represented an aggregate with an unacceptable durability factor and a high expansion, but passed the soundness test with a good rating. Section 4 represented an extremely poor aggregate with a very low Durability Factor, high expansion and a soundness rating. This section represented failing the new specifications, but would have been acceptable under the old specifications. Original test results and results from the 1997 inspection are shown in Table 2. It is quite apparent that the current specification limits are quite acceptable and lowering the limits would increase the risk of pavement deterioration.

Table 2. US 169, Johnson County, Kansas Pavement Inspection Results. 1997

Date Placed	Section	Durability Factor (C 666)	Expansion (C 666)	Freeze-thaw (Soundness)	Amount Joint Deterioration (% of joints)	Type Deterioration
5/82	1	96	0.018	0.96	0	NA
4/81	2	95	0.024	0.88	0	NA
12/81	3	78	0.084	0.98	93	Spalling
5/82	4	56*	0.118	0.93	60	D-cracking

* 284 cycles

Recent Experience

After several years of study the overall mechanism by which D-cracking is initiated is better understood. The Kansas Department of Transportation has been well served by the specifications implemented in 1981 as represented by Figure 2.

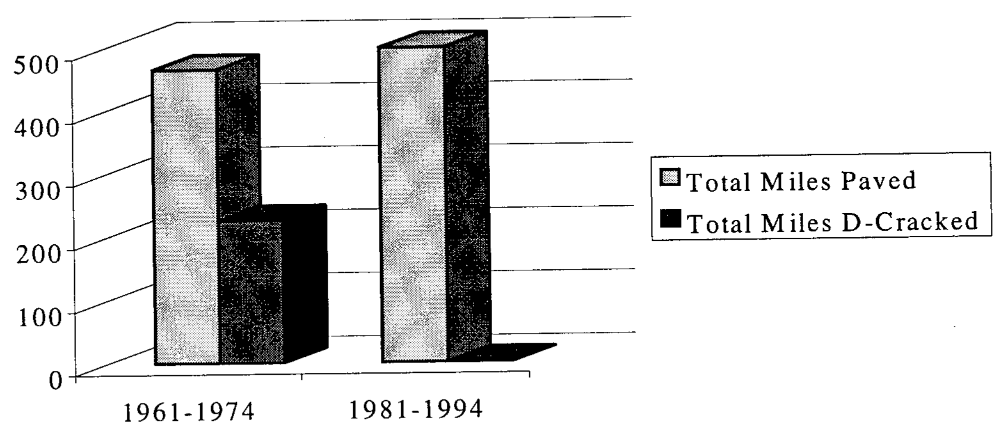

Figure 2. Comparison of Paved Miles and D-cracked Pavements.

The results from the 1980 field study revealed that approximately 48% of the pavements constructed between 1961 and 1974 were D-cracked. Since 1981 and the adoption of the current specifications, KDOT has constructed 465 miles of PCCP. To date only 0.8 km (0.5 miles)[2] have active D-cracking according to the 1997 Pavement Management Survey [20]. An additional 10 miles of stained pavement that may be in the preliminary stages of D-cracking was also reported.

Current Practice

Today, KDOT pavements are relatively free of D-cracking. The Class VI aggregate is no longer an accepted aggregate classification. In the beginning of the 1980 study, Class I aggregate had yet to be identified. The Class VI classification was a temporary classification designed for initial acceptance of an aggregate until the C 666 test was completed. Most limestone aggregate ledges from active quarries have been tested several times over the years and it is reasonably well understood which limestones will most likely pass the Class I testing. Class VI classification is no longer used by KDOT.

Active quarries are inspected approximately every two years and KDOT geologists determine when an aggregate ledge needs to be re-evaluated. The Materials and Research Laboratory in Topeka receives "as quarried" ledge samples which are crushed and tested according to KT-MR-21, *Soundness and Modified Soundness of Aggregates by Freezing and Thawing Test Method.* The test method is included as Appendix A in this report. If the ledge sample passes the modified soundness test, it is then subjected to specific gravity, absorption, acid insoluble residue and LA Wear and the modified[3] ASTM C 666, Procedure B.

Aggregates not meeting the Class I classification may not be used in PCCP although the aggregates may still be suitable for use in many other concrete and asphalt applications. Appendix B contains a complete listing of KDOT's Specifications on Aggregates for Concrete.

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[2] Aggregate in the D-cracked sections is from one quarry

[3] As previously stated KDOT uses ASTM C 666, Procedure B modified to include a 90-day cure before the freezing and thawing portion of testing.

REVIEW OF KDOT RESEARCH ON ALKALI-SILICA REACTIVITY (ASR)

Suitable coarse aggregates for use in concrete are not widely available in western Kansas, because of this it became common practice to construct pavements and structures with total sand-gravel aggregates. The term sand-gravel is used to describe the total aggregate produced from the deposits of sand and gravel found in the riverbeds and valleys of local rivers. The sand-gravel used as aggregates were found chiefly in the Kansas, Arkansas, Republican and Blue Rivers and their tributaries. These aggregates were siliceous and contained 5% to 15% limestone particles. Opal and other similar expansive minerals were not present in any significant amount (1%). The concrete produced with these aggregates in general showed satisfactory properties and generally had good service. The first concrete pavement in Kansas, which developed typical map cracking due to ASR, was constructed in 1927-1928. Bad map cracking and deterioration had developed by 1931 [21]. The deterioration could also be found in bridges and could easily be detected when the concrete began to dry after a light rain. The same type deterioration was also found in Iowa, Nebraska and Missouri [23].

Due to the alarming rate of deterioration, research was instigated [21] in early 1932 and concentrated on recreating the deterioration in the laboratory to possibly determine its cause and cure. Concrete specimens were subjected to different curing conditions and cycles of freezing and thawing but no cracking occurred. The specimens were then subjected to a series of wetting and drying which did re-create the deterioration. Conclusions from the first study were that the map cracking could not be predicted by chemical analysis of the cement. Also, map cracking occurred predominately in total sand-gravel mixes; and the laboratory results obtained through a series of wetting and drying cycles correlated almost perfectly with the field service records of the aggregates.

Development of the Wetting & Drying Test

Between the years 1934 and 1941, the Kansas State Highway Commission and the Engineering Experimental Station at Kansas State University made a number of field studies into the durability of aggregates [21, 22]. The purpose was to investigate the relationship between the types of failures and the materials incorporated into the concrete. In 1942, the Portland Cement Association entered an agreement with the Engineering Experiment Station and the

Kansas Highway Commission to execute a cooperative research program. The original investigation [23, 24] was divided into three major parts.

1. A Pilot Series to develop an accelerated laboratory test that would produce expansion and cracking similar to that observed on pavements and structures in service.
2. A Cement Series to determine the chemical or physical characteristics of the cement that might contribute to the expansion or cracking.
3. A Curative Series to study various means of preventing the expansion and cracking.

Pilot Series

The Pilot Series outlined the work and served as a guide in:

(1) Developing and selecting an accelerated method to test.

(2) Determine the best method of measuring and characterizing the deterioration produced in the test.

(3) Establish the significance of certain variables in the cements and aggregates to be used in conducting the investigation.

(4) Establish suitable test limits for the conduct of the rest of the investigation.

Materials chosen consisted of nine brands of Portland cement each mixed into four different blends to provide alkali contents ranging from 0.23 to 0.94% calculated as Na_2O equivalent. The aggregates were chosen based on their field performance varying from very good to very unsatisfactory. Concrete beams 7.8 x 10 x 40 cm (3 x 4 x 16 in.) were constructed, cured following the same procedure and submitted to seven types of exposure. The seven types of exposure studied in the pilot series were:

Exposure 1: A modified version of a wetting and drying exposure method developed by Guy O. Gardner of Ash Grove Cement in Chanute, Kansas. Following the 30-day curing period, the beams were immersed in water at 40°C (113°F) for three days. They were then immersed in water at 21-27°C (70-80°F) for 28 days and then placed in an air-conditioned room for seven days. The beams were then placed in a drying oven set at 49°C (120°F) for seven days, following which they were returned to water storage at 21-27°C (70-80°)F until the conclusion of the test.

Exposure 2: An alternate wetting and drying cycle was developed by W. E. Gibson in an earlier study [21]. After the 30-day cure, specimens were subjected to 16 hours in water at 21°C (70°F) and eight hours in an oven at 52°C (125°F) repeated daily except for Sundays.

Exposure 3: A continuous storage in warm, moist air in closed containers at approximately 49°C (120°F). (Specimen temperatures were about 44°C (112°F).

Exposure 4: Continuous storage in a dry air oven at 49°C (120°F).

Exposure 5: Continuous storage in the laboratory at room temperature of 16-38°C (60-100°F) and 30-80% relative humidity.

Exposure 6: Continuous moist room storage at 21°C (70°F) and 95% relative humidity (minimum).

Exposure 7: Outdoor storage for natural weathering.

Results of the Pilot Series found that of the seven types of exposure tested exposure 2, the alternating wetting and drying cycle, proved to be the most discriminating. Results of the accelerated test closely paralleled field service records. It was determined that the Cement Series and the Curative Series would be studied under four of the exposures. The wetting & drying conditions (exposure 2), standard moist room storage at 21°C (70°F) (exposure 6) and 54°C (130°F) (exposure 3), and natural weathering conditions (exposure 7).

Cement Series

In the Cement Series, 22 cement samples commonly used in the test region along with two samples from outside the test region were collected. Each cement specimen was tested with two different sand-gravel aggregates, one representing a good service record and the other representing a poor service record.

Beams identical to those made in the Pilot Series were molded for each cement aggregate combination. The beams were then subjected to the four exposures described. Results of the cement series indicated no difference in deterioration between cements of different chemical analysis.

Curative Series

This series consisted of a study of various means of eliminating or minimizing the expansion and map cracking experienced with sand-gravel concrete. Measures studied were:

1. Coarse aggregate additions.
2. Additions of powdered materials passing the #200 sieve, including inert and pozzolanic materials.

3. Air-entrained concretes.
4. Chemical additions to the concrete mix.
5. Addition of waterproofing agents.
    (a) Red oil
    (b) Aluminum stearate
    (c) Raw linseed oil
6. The effect of surface area and SO_3 content of the cement.
7. Miscellaneous.

After evaluation of 10 years of outside field exposure and the wetting and drying test results, conclusions reported [25] were:

1. The wetting and drying test results provided a good correlation with field service.
2. The addition of crushed limestone in amounts of 25% to 50% by weight of the total aggregate was effective in reducing the expansion to such an extent that concrete will be produced which will render satisfactory service.
3. The addition of mine-run or screened chats in amounts not less than 25 percent by weight of the total aggregate was effective in reducing cement-aggregate reaction.

In the course of the investigation, it was found that the addition of certain pozzolanic materials might be helpful in suppressing the cement-aggregate reaction observed. Air-entrained concretes almost invariably were slower in developing the reaction in the laboratory tests than were the non air-entrained concretes. However, the difference was not significant. Certain chemical additions gave some indication of being helpful, but were not adequate to inhibit deterioration in 10 years. The addition of waterproofing agents (red oil, aluminum stearate, and raw linseed oil) were helpful in slowing down the rate of reaction, but were not successful in eliminating the effect.

A study of the effect of surface area and SO_3 content of the cement indicated that somewhat higher SO_3 contents than allowed under current specifications would slow down and somewhat suppress the reactions observed.

Near the end of this investigation, it was evident that the wetting and drying test method was a valid means of qualifying sand gravel aggregates for Kansas pavements and structures. The test method was published by Scholer [26] in the ASTM Proceedings of 1949.

The McPherson Test Road

In 1949, based on the results of the previous study, the Kansas State Highway Commission, the U.S. Bureau of Public Roads, the Portland Cement Association, and other agencies constructed a six-mile experimental test road in McPherson, Kansas [27, 28, 29, 30]. The objective was to explore the possibilities of inhibiting map cracking and abnormal expansion by addition of various sand-gravel aggregates in combinations with different cements, limestone sweetening and pozzolanic admixtures [22].

The McPherson Test road, part of US 81, was built from sand-gravel aggregates produced from the Republican River, one of the most reactive aggregates in Kansas. The test pattern consisted of three sections. One test section used basic sand gravel, one section had a 30 percent replacement of the aggregate with crushed limestone, and the third section used the basic aggregate with a portion of the cement replaced by one of three pozzolanic materials. This pattern was repeated with five brands of cement (four Type I and one Type I/II), and with and without air entraining.

Semi-annual surveys and testing were conducted for a ten-year period [29, 30]. Sources of the data were:

1. Test beams
2. Surveys of pavement condition
3. Soniscope readings of the slab
4. Soniscope readings of test beams
5. Strain gage readings for measurement of volume change
6. Measurement of faulted joints
7. Count of panels pumping and blowing up
8. Count of mudjacked panels
9. Measurement of test beam deflections
10. Weather records
11. Traffic Data
12. Construction notes
13. Initial reports
14. Durability tests by freezing and thawing.

A published report on the status of the road was made at the end of 10 years [30]. The following observations with respect to map cracking, flexural strength, expansion, transverse and longitudinal cracking, and Soniscope pulse velocities were made.

Development of Map Cracking

None of the pozzolans or limestone sweetening was completely successful in preventing map cracking. In addition, only limestone sweetening was effective in reducing map cracking at all stages in the test and in delaying its initial appearance. With all cements used and with both non-air-entrained and air-entrained sections, limestone sweetening gave a marked improvement over basic sections and was more effective than any of the pozzolans.

There was a decided reduction in map cracking with the Type II cement, but it alone did not arrest map cracking.

Map cracking appeared about one year earlier in the air-entrained sections and initially the cracking developed faster than non-air-entrained sections. However, by the end of 10 years cracking in the air-entrained and non-air-entrained sections were increasing at the same rate.

Flexural Strength

Beams were tested twice a year for flexural strength. Test beams containing only the basic aggregate lost strength steadily throughout the test period. The limestone-sweetened beams were uniform over the 10-year period. The beams with pozzolanic materials were very strong and gained strength through the test period. However, they had a tendency toward wide variations in strength from spring to fall, with the fall breaks yielding only one-half the values obtained in the spring. This tendency was not true for limestone sweetened beams where the seasonal differences was small and often even reversed.

Flexural strength of test beams in relation to the various cements showed that the Type I cements were all similar at all ages. The Type II cement gained strength by comparison and at the end of the 10-year period was about 175 psi stronger than the others.

Test beams representing air-entrained sections averaged 13% lower strengths than their non-air-entraining counterparts.

Expansion

All of the tested pozzolans and the limestone sweetened concretes prevented excessive expansion. Also, there was little evidence of expansion with regard to the various cements and although the amount of expansion was not great, the relative advantage of the Type II cement was quite evident. It was also evident that expansion was reduced in the air-entrained sections.

Transverse and Longitudinal Cracking

The pozzolans produced concrete of high strength but the pavements were subject to large numbers of structural cracks. The limestone-sweetened sections were free of longitudinal cracks and tended to crack less transversely. Again, the Type II cement proved to produce less cracking overall when compared to the Type I cements.

Soniscope Pulse Velocities

A high pulse velocity was thought to be indicative of good concrete quality and a low velocity to indicate a poorer quality. Following the test period, evaluation of results showed that a change in pulse velocity was indicative of a quality change, although the absolute value was not in itself a measure of concrete strength.

Conclusions

1. All of the tested pozzolans and limestone sweetening prevented excessive expansion although none of the combinations eliminated the cracking. The depth of the cracking and the pattern tended to change with these additives. Limestone sweetening was the most effective in reducing map cracking.
2. Concrete made with pozzolans attained higher flexural strength that increased with age, but was susceptible to seasonal fluctuations.
3. The pozzolans used with the Type I cements had more transverse and longitudinal cracks. The Type II cement also had cracking, but on a lesser scale.
4. The quality of the concrete varied widely dependent on the aggregate and cement source. Concrete made with the Type II cement developed less map cracking regardless of the aggregate type.
5. Concrete made with the Type II cement gained flexural strength at a slower rate, but have remained higher than the other cements after 2½ years.
6. Air-entraining did not inhibit or increase map cracking.
7. Air-entraining developed lower flexural strength than the non-air-entraining counterparts.
8. Air-entraining developed more transverse and longitudinal cracking than the non-air-entrained concretes.

Recommendations

Based on the results of the study the following recommendations were made:

1. Kansas should continue to use limestone sweetening with sand-gravel of dubious quality.
2. Using cements with total alkali less than 0.60% can provide some protection from map cracking with sand gravel aggregates.
3. Further research to determine why some cements perform better than others in regard to map cracking. Also, further research into the use of pozzolans.

The knowledge gained from these studies has been invaluable. The study of concrete from the 1942 study and the McPherson Test Road continued for several years and resulted in several published reports by some of the most notable experts of the time [31, 32, 33, 34].

Current Practice

Since the McPherson test road project, limestone sweetening and the use of low alkali cement has been a common practice in Kansas to control map cracking and premature deterioration. It has proven to be highly effective in eliminating map cracking and ASR in Kansas PCC pavements.

KDOT specifications require that before a naturally occurring, predominantly siliceous aggregate can be used as a total concrete aggregate it must pass the Wetting and Drying Test of Sand and Sand Gravel Aggregate For Concrete. The test determines the acceptability of sand gravel aggregate to be used in concrete for the construction of pavements, bases, bridges, flumes, slope drains, sidewalks, riprap, curb and gutter, building floors, ramps to buildings, parking lots, wash checks and ditch linings.

Briefly, the test provides preparing from a standard MA-1 aggregate gradation concrete beams (7.8 x 10 x 4 cm) in a standard pavement mix design with a standard laboratory cement. The beams are cured for 30 days in a moist room. The beams are then placed in a water bath maintained between 16-27°C (60-80°F) for 16 hours followed by oven drying at 53-54°C (128-130° F) for eight hours. The procedure is repeated daily Monday through Friday for a one-year period. On weekends and holidays, the specimens remain immersed in the water bath. Approval of the aggregate is based on two criteria, expansion and modulus of rupture. The specimens are measured for expansion every month for one year and the modulus of rupture measured at age 60 days and one year. Acceptable limits are a maximum expansion of 0.05% at 180 days and 0.07% at 365 days. The modulus of rupture must be at least 3.8 MPa (550 psi) for both measurements. A complete description of the test method is included in Appendix C.

References

1. Scholer, C. H. 1928. *Some Accelerated Freezing and Thawing Tests on Concrete,* Proceedings of the American Society for Testing Materials, Vol. 28:472-486.

2. Gibson, W. E. 1941. *Significance of Soundness Tests of Aggregates*, Highway Research Board, Vol. 21:283-287.

3. Scholer, C. H., and A. E. Stoddard. 1932. *Proposed Method of Testing Concrete and Concrete Aggregates by Freezing and Thawing,* Proceedings of the American Society for Testing Materials, Vol. 32:364-365.

4. Scholer, C. H. September 14, 1944. Written Communication to R. D. Finney, Engineer of Materials, Kansas State Highway Commission.

5. Tiffany, Geo. E. Engineer of Construction, Kansas Highway Commission, *Memorandum on Concrete Pavements,* Internal Report to R. C. Keeling, State Highway Engineer, Kansas Highway Commission, December 5, 1944.

6. Kansas Highway Commission. March 1945. *A Discussion of Concrete Pavements in Kansas*, Unpublished Internal Report.

7. Finney, R. D. July 1946. *Resume of Research Studies Conducted by the Kansas Highway Commission Over the Past Fifteen Years,* Internal Report, Kansas Highway Commission.

8. White, L. V., and R. L. Peyton. 1945. *Condition of Concrete Pavements in Kansas as Affected by Coarse Aggregate,* Highway Research Board Proceedings, Vol. 25:129-146.

9. Peyton, R. L. 1954. *Research in Concrete Mixtures and Concrete Pavements in Kansas,* Internal Report, Kansas Highway Commission.

10. Bukovatz, John E., Carl F. Crumpton and Herbert E. Worley. 1973. *Study of D-cracking in Portland Cement Concrete Pavements, Volume 1, Field Phase,* Kansas Department of Transportation.

11. Bukovatz, John E., and Carl F. Crumpton. 1981. *Study of D-cracking in Portland Cement Concrete Pavements, Volume 2, Laboratory Phase.* FHWA-KS-81-2, Kansas Department of Transportation.

12. Myers, Lewis D., and Alvis H. Stallard. 1978. *Study of D-cracking in Portland Cement Concrete Pavements, Volume 3, Air Photo Phase*, FHWA-KS-78-2, Kansas Department of Transportation.

13. Crumpton, Carl F., John Wojakowski, Harvey E. Wallace and L. Dean Hamilton. 1994. *Study of D-cracking in Portland Cement Concrete Pavements, Volume 4, Petrographic Phase and Final Report*, FHWA-KS-94-3, Kansas Department of Transportation.

14. Best, Cecil H. 1974. *D-cracking in PCC Pavements – Cause and Prevention,* Final Report No. KSU-EES-2472.

15. Wallace, Harvey E., and L. Dean Hamilton. 1982. *An Investigation of Kansas Limestone as They Pertain to the D-cracking Phenomena.* Internal report, Kansas Department of Transportation.

16. Wallace, Harvey E. *An Updated Report on D-cracking of Concrete Pavements in Kansas,* Proceedings, National D-cracking Workshop, 1990.

17. Bisque, R. E., and J. Lemish. 1958. *Chemical Characteristics of Some Carbonate Aggregate as Related to Durability of Concrete,* Highway Research Board Bulletin No. 196:29-45.

18. *1980 Standard Specifications for State Road and Bridge Construction*, Kansas Department of Transportation, pg. 1102.

19. Special Provision 80P-103, Kansas Department of Transportation, April 1981.

20. Clark, G. Norman. 1997. *Pavement Management System, 1997 NOS Condition Survey,* Kansas Department of Transportation.

21. Gibson, William E. 1938. *A Study of Map Cracking in Sand-Gravel Pavements,* Proceedings, Highway Research Board, Vol.18:227-237.

22. Scholer, Charles H., and William E. Gibson. 1948. *Effect Of Various Coarse Aggregates Upon the Cement-Aggregate Reaction,* Proceedings, Journal of the American Concrete Institute, Vol. 19, No. 10, pp. 1009-1032, 1948.

23. Scholer, Charles H., and Phil J. Tatman. 1949. *A Study of Special Concrete Problems in Kansas, Nebraska, Iowa and Western Missouri, Third Progress Report,* Kansas Highway Commission.

24. Scholer, Charles H. 1955. *A Study of Special Concrete Problems in Kansas, Nebraska, Iowa and Missouri, Summary Report,* Kansas Highway Commission.

25. Peyton, R. L. 1954. *Research in Concrete Mixtures and Concrete Pavements in Kansas,* Internal Report, Kansas Highway Commission.

26. Scholer, Charles H. 1949. *A Wetting and Drying Test for Predicting Cement-Aggregate Reaction.* Reprint, Proceedings of the American Society for Testing Materials, Philadelphia, Pa., Vol. 49.

27. State Highway Commission of Kansas, Research Department. 1954. *Effect of Pozzolans Added to Sand Gravel Concrete Pavement in a Kansas Test Road.*

28. Peyton, R. L. 1950. *Report on Condition of Experimental Project, McPherson, Kansas,* Internal Report, Kansas Highway Commission.

29. Lovewell, C. E. 1953. *Report on the Condition Survey of the McPherson, Kansas Test Road,* Internal Report, Kansas Highway Commission.

30. Stingley, W. M., John D. McNeal and R. L. Peyton. 1960. *The McPherson Test: Tenth-Year Report,* Highway Research Board Proceedings, Vol. 39:191-204.

31. Gibson, W. E. 1955. *Cement Aggregate Reaction in Concrete Showing the Correlation Between Laboratory Tests and Outside Weathering,* Internal Report, Road Materials Laboratory, Kansas State Highway Commission.

32. Lerch, William. 1959. *A Cement-Aggregate Reaction that Occurs with Certain Sand-Gravel Aggregates,* Journal of the PCA Research and Development Laboratories.

33. Hadley, David W. 1968. *Field and Laboratory Studies on the Reactivity of Sand-Gravel Aggregates,* Journal of the PCA Research and Development Laboratories, January 1968.

34. Mather, Katherine, and Richard C. Mielenz. 1960. *Cooperative Examination of Cores from the McPherson Test Road,* Proceedings, Highway Research Board, Vol. 39:205-215.

APPENDIX A

KTMR-21 SOUNDNESS & MODIFIED SOUNDNESS OF AGGREGATES BY FREEZING AND THAWING

a. SCOPE

This test method covers the procedures for the determination of aggregate resistance to disintegration by freezing and thawing. For the pre-qualification of quarry ledge rock and for verification after crushing production begins.

b. REFERENCED DOCUMENTS

b.1. AASHTO M 92; Wire-Cloth Sieves for Testing Purposes

b.2. AASHTO M 231; Balances Used in the Testing of Materials

b.3. AASHTO T 103; Soundness of Aggregates by Freezing and Thawing

c. APPARATUS

c.1. A freezing tank capable of maintaining a denatured alcohol bath at -32.2 to -17.7°C (-20 to 0°F).

c.2. The sample containers shall be of non-corroding metal cylindrical in shape approximately 114 mm in diameter by 559 mm in length (4.5 X 22 in).

c.2.a. A rack capable of containing eight sample containers at one time.

c.3. Sieves shall conform to the requirements of AASHTO M 92.

c.4. A balance of sufficient capacity, be readable to 0.1% of the sample mass, or better, conforming to the requirements of AASHTO M 231.

c.5. The drying oven shall provide a free circulation of air through the oven and shall be capable of maintaining a temperature of 110 ± 5°C (230 ± 9°F).

c.6. A thawing tank of suitable size to hold the samples and containers, allowing total submersion of the samples. The water shall be maintained at 21-27°C (70-80°F) during the thaw period.

d. SAMPLE PREPARATION[4]

d.1. Class 1 Aggregate (Modified Soundness)

d.1.a. Preliminary preparation may require crushing the material to pass the 19 mm (3/4 in) sieve. Remove all material retained on the 19.0 mm (3/4 in) mesh sieve and that passing the 9.5 mm (3/8 in) sieve, and remove all mud, clay lumps or sticks. The material shall not be washed. Shale, shale-like material, coal, asphalt coated pieces, rotten stone, soft or friable particles and other foreign material

[4] There are two classes of aggregate Class 1 Aggregate ($^3/_4$-$^3/_8$ in) and Official Quality Aggregate ($^3/_4$ in - # 8).

shall **not** be removed prior to testing. The material shall then be oven dried to a constant mass at a temperature of 110 ± 5°C (230 ± 9°F).

d.1.b. Final preparation shall consist of screening the oven dried material over 19.0, 12.5, 9.5 mm (3/4, 1/2, and 3/8 in) sieves to meet the following grading.

Table A1. Gradation Requirements for Modified Soundness Test

Individual Sizes	Cumulative Mass Retained (grams)
19.0 mm (3/4 in)	0
12.5 mm (1/2 in)	2,500
9.5 mm (3/8 in)	5,000

d.2. Official Quality Aggregate (Soundness)

d.2.a. Preliminary preparation shall include the removal of all material passing the 2.36 mm (No. 8) sieve, mud or clay lumps and sticks. Shale or shale-like materials, coal, asphalt coated pieces, rotten stone, soft or friable particles and other foreign material shall **not** be removed prior to testing. Any pieces retained on the 25.0 mm (1 in) sieve shall be removed, crushed to pass the 25.0 mm (1 in) sieve, and all pieces larger than the 2.36 mm (No. 8) sieve returned to the sample. The material shall then be soaked for a period of 24 ± 4 hours.

d.2.b. Final preparation shall consist of bringing the sample to a surface dry condition at room temperature, screened over 25.0, 19.0, 9.5, 4.75, 2.36 mm (1 in, 3/4 in, 3/8 in, No. 4, and No. 8) sieves and a 5,000 gram test sample selected to meet one of the gradings shown below.

Table A2. Gradation Requirements for Soundness Test

Grading Designation	Cumulative Mass Retained (grams) Individual Square Mesh Sieves				
	25.0 mm (1 in)	19.0 mm (3/4 in)	9.5 mm (3/8 in)	4.75 mm (No. 4)	2.36 mm (No. 8)
I	0	2250	1750	500	500
II	...	0	3500	1000	500
III	...	...	0	4000	1000

Note a: The gradings shown in the table are for crushed material and the grading selected will be contingent upon the maximum aggregate size in the sample. The grading of the test sample shall be the same (± 5%) as the material submitted for this test. The largest particle size appearing in the test shall be that size which is present in the "as received" material equal to or in excess of 2%.

d.2.c. Official Quality Sand and Gravel

d.2.c.1. For sand-gravel, a sieve analysis of the plus 2.36 mm (No. 8) "as received" material shall be determined. A 5000 g sample based on the "as received" gradation will be used for the soundness test.

e. PROCEDURE

e.1. After sieving, the material shall be placed in an open top container, covered with a 1.18 mm (No. 16) mesh cloth and submerged in tap water maintained at a temperature from 21-27°C (70-80°F).

e.1.a. For Class 1 Aggregates soak for a period of 24 ± 4 hours.

e.1.b. For Official Quality Aggregates, including Official Quality Sand-Gravel, soak for 1 - 2 hours.

e.2. Remove sample from the water, drain, and while in a saturated and drained condition place in the freezing equipment that maintains a temperature between -29°C and -18°C (-20°F and 0°F). The sample remains in the freezing equipment until frozen, but in no case shall this period of time be less than two hours. During any interruptions (nights, weekends, and holidays) the samples remain in the freezer.

e.2.a. Remove sample from the freezer and place in a tap water bath maintained at 21-27°C (70-80°F) for a period of 40 minutes.

e.3. One freezing period and one thawing period shall be considered one cycle. After the sample has been subjected to 25 cycles of freezing and thawing, wash over a 1.7 mm (No. 12) sieve.

e.3.a. For Class 1 Aggregates, oven dry samples to a constant mass at a temperature of 110 ± 5°C (230 ± 9°F). Screen sample over a 12.5 and 9.5mm (1/2 and 3/8 in) mesh sieve and record the mass.

e.3.b. For Official Quality Aggregate and Sand-Gravels, surface dried samples are screened over the 25.0, 19.0, 9.5, 4.75, and 2.36 mm (1 in, 3/4 in, 3/8 in, No. 4, and No. 8) sieves and record each mass.

f. CALCULATIONS

f.1

$$LR = \frac{A}{B}$$

Where: LR = freeze-thaw loss-ratio

A = the cumulative mass of the sample at the end of the test

B = the cumulative mass of the sample at the beginning of the test

g. INTREPRETATION OF RESULTS

g.1. Class 1 aggregates must have a LR of 85 or better to be tested under KTMR-22.

g.2. Class 1 aggregates reporting less than 85 are designated Class 0.

APPENDIX B

The following specifications are draft versions. They have been updated from the Kansas Department of Transportation 1980 Standard Specifications for State Road and Bridge Construction for the KDOT 2000 Standard Specifications book.

SUBSECTION 1101

GENERAL REQUIREMENTS FOR AGGREGATES

1101.01 DESCRIPTION

This specification covers the basis of approval, certification and acceptance of aggregates specified in section 1100.

1101.02 REQUIREMENTS.

a. General.

Provide aggregates that meet all composition, quality, product control, and handling (stockpile) requirements of the applicable specifications.

b. Process Control.

(1) Perform or cause to be performed all inspections and tests necessary to provide and maintain an adequate process control system that will provide reasonable assurance that all aggregates or aggregate combinations submitted for acceptance will conform to contract requirements.

Prior to beginning aggregate production, submit a proposed Process Control Plan in writing for review by the Engineer or the QC/QA Contractor. Include the sampling and testing frequencies, the sampling locations, the sampling and testing methods, and other inspections expected to establish and maintain process control in the plan. If requested, the Department will make a chart of recommended sampling and testing frequencies for process control available to the Producer.

A process control plan should include procedures for all aggregates produced to determine gradation, plasticity index, deleterious substance content, and other properties that may be required by the specification, and to inspect stockpiles for separation, contamination and segregation. These guidelines are considered normal activities necessary to control the production of aggregates or aggregate combinations at an acceptable quality level. It is recognized that, depending on the type of process or materials, some of the activities listed may not be necessary, or other activities may be required. The frequency of these activities is not listed in these guidelines as they will vary with the process and the materials.

(2) Sampling and Testing.

Use the same sampling and testing methods and procedures in process control as those used by the Department. The Department will make the Kansas Test (KT) Methods that are approved for Department use available to the Producer.

(3) Test Reports.

Maintain a file of all process control tests. Furnish copies to the Engineer upon request.

(4) Acceptance Inspection.

Acceptance of aggregate will be based on Department and/or Contractor tests at the point of usage unless designated otherwise by the Engineer. Aggregate production will also be inspected to determine if aggregates are being produced from deposits, ledges, and beds which meet the specific quality requirements. Aggregates produced from deposits, ledges, or beds that have not been previously approved for quality will be rejected. Remove rejected material from the project stockpile area immediately. Any work incorporating aggregates from sources not approved for quality for that work must be removed and replaced, or otherwise corrected, by and at the expense of the Contractor.

The Department reserves the right to run any test at any time to determine specification compliance. When test results on aggregates or mineral filler supplements indicate non-compliance with specifications, the Engineer may cause those materials to be rejected and removed from the worksite at the expense of the Contractor.

c. Certification of Aggregates.

Provide the Engineer a certification for each classification of aggregate utilized in a project.

(1). <u>Aggregates Delivered to the Site</u>: Certify each classification of aggregate delivered to a project or product preparation site. Prepare these certifications under the signature of the aggregate producer or their designated representative.

(a) Certify aggregates that are tested at their destination to determine final disposition as to the locations of the deposits from which they were produced.

(b) Certify aggregates that are tested at their production site to determine final disposition. These certifications state that the aggregates were removed from a Department tested and approved stockpile at the production site, or that they were removed from a plant while it was producing aggregate that was in compliance with the applicable specifications.

(2) <u>Aggregates Incorporated into the Project</u>: At locations where aggregates and products that incorporate aggregates are produced for Department **and** non-department use, provide certifications stating that only Department tested and approved aggregate was provided for the Department projects.

(3) <u>Frequency of Certification</u>:

(a) Prior to the initial delivery of aggregates to a project or product preparation site, provide the Engineer a certification from. This certification is to be under the signature of the aggregate producer or their designated representative and state that all aggregates to be provided for the project are in compliance with all the applicable Department specifications.

(b) Upon completion of the project, provide certifications as specified in **1101.02c.**(1) and **1101.02c.**(2) of this specification to the Engineer. These certifications are to apply to all aggregates that were delivered to the project or product preparation site and ultimately used in the project.

These certifications are to indicate the approximate quantities in tons or cubic meters of each aggregate delivered to the project and the approximate quantities in tons or cubic meters of each aggregate delivered to the product preparation site and incorporated into a product that was utilized in the project.

1101.03 TEST METHODS

Test all aggregates in accordance with the applicable methods cited in subsection 1117.

1101.04 PREQUALIFICATION

Aggregates from each source require "Official Quality" testing on samples obtained by an authorized representative of the Department prior to use on Department projects. These samples are taken from actual production, which may be "pit-run", "crusher-run" or which may involve some processing. Approved sources remain approved only if there are no major changes in the production methods or deposit characteristics.

1101.05 BASIS OF ACCEPTANCE

Aggregates from sources approved for the intended use are accepted based on the following:

a. Current official quality test results meeting the requirements of the applicable subsection are on file with the department.

b. Results of tests conducted on samples taken at a point or points designated by the Engineer. The Department reserves the right to re-sample, test and reject any previously accepted aggregate if the Engineer has reason to believe it no longer meets all requirements of the contract.

c. Certifications as specified above.

SUBSECTION 1102

COARSE AGGREGATES FOR CONCRETE

1102.01 DESCRIPTION.

This specification covers coarse aggregates for use in all types of concrete construction.

1102.02 REQUIREMENTS.

a. Composition.

(1) Coarse Aggregate may be crushed or uncrushed gravel, chat, or crushed stone. (Consider limestone, calcite cemented sandstone, ryolite, basalt and granite as crushed stone).

b. Quality.

(1) The requirements for Coarse Aggregate for Structures and Non-grade applications are found in Table 1102-1.

TABLE 1102-1

Quality Requirements for Coarse Aggregates for Structures & Non-grade Applications

Concrete Class	Soundness[1] (min.)	Wear (max.)	Absorption (max.)	Acid Insol. (min.)
Grade xx (AE)(SW)[2]	0.90	40	-	-
Grade xx (AE)(SA)[3]	0.90	40	2.0	-
Grade xx (AE)(AI)[4]	0.90	40	-	55
Grade xx (AE)(PB)[5]	0.90	40	3.0	-
All Others	0.90	50	-	-

[1] Soundness will be waived if the aggregate meets all requirements for Class I aggregate.
[2] Grade xx (AE)(SW) - Structural concrete with select coarse aggregate for wear.
[3] Grade xx (AE)(SA) - Structural concrete with select coarse aggregate for wear and absorption.
[4] Grade xx (AE)(AI) - Structural concrete with select coarse aggregate for wear and acid insolubility.
[5] Grade xx (AE)(PB) - Structural concrete with select aggregate for use in prestressed concrete beams.

(2) Coarse Aggregate for Concrete on Grade. (Soundness will be waived if the aggregate meets all requirements for Class I aggregate)

(a) All aggregates except limestone
- Soundness, minimum 0.90
- Wear, maximum...50%

(b) Additional Requirements for Crushed Limestone or Dolomite (Class I aggregate)
- Modified Soundness, minimum 85%
- Durability Factor, minimum...95
- Expansion, maximum..0.025%

(3) Coarse Aggregate for Bridge Deck Wearing Surface.

- Soundness, minimum.. 0.95
- Wear, maximum...40%
- Acid Insoluble Residue, minimum......................................55%

c. Product Control

(1) Size requirements. Provide aggregate that complies with the requirements shown in Table 1102-2.

TABLE 1102-2 Requirements for Coarse Aggregates for Concrete

Type	Usage	Composition	Percent Retained - Square Mesh Sieves							
			37.5 mm	25 mm	19 mm	12.5 mm	9.5 mm	4.75 mm	2.36 mm	600 µm
CA-1	Pavement only [1]	Siliceous Gravel or Crushed stone except Limestone or Dolomite	0	0-10	30-65	-	70-93	-	95-100	-
CA-3	All concrete	Chat	0	0-5	-	-	-	55-75	87-97	95-100
CA-4	All concrete	Siliceous Gravel or Crushed Stone	-	-	0	0-30	30-60	75-100	95-100	-
CA-5	All concrete except pavement	Siliceous Gravel or Crushed Stone	-	0	0-5	-	40-60	-	95-100	-
CA-6	Use with Basic Aggregate	Siliceous Gavel, Chat or Crushed Stone	-	-	0-5	-	-	-	95-100	-
CA-7	Bridge Deck Wearing Surface	Siliceous Gravel, Chat, or Crushed Stone	-	-	0	0-10	15-50	85-100	-	-

[1] CA-1 may be used for other work that is part of a concrete pavement contract.

(2) Deleterious Substances

Maximum allowed deleterious substances by mass are:

- Material passing the 75 µm sieve.................................... 2.5%
- Shale or Shale-like material.. 0.5%
- Soft or friable particles... 2.5%
- Sticks (wet)..0.1%
- Coal.. 0.5%
- Clay lumps..0.5%

None of the individual components listed can be exceeded. Combinations excluding the 75 µm requirement cannot exceed 3.0%.

(3) Uniformity of Supply
Designate or determine the fineness modulus (gradation factor) in accordance with the procedure listed in the Construction Manual Part V, Section 17 prior to delivery or from the first 10 samples tested and accepted. Provide aggregate that is within ±0.20 of the average fineness modulus.

d. Handling Aggregates

(1) Segregation.
Remix all aggregate that has segregated through transit or stockpiling before acceptance testing.
(2) Stockpiling.

(a) Stockpile accepted aggregates in layers 1.0 to 1.5 meters thick. Berm each layer so that aggregates do not "cone" down into lower layers.
(b) Keep aggregates from different sources, with different gradings or with a significantly different specific gravity separated.
(c) Transport aggregate in a manner that ensures uniform gradation.
(d) Do not use aggregates that have become mixed with earth or foreign material.
(e) Stockpile or bin all washed aggregates produced or handled by hydraulic methods for 12 hours (minimum) before batching. Rail shipment exceeding 12 hours is acceptable for binning provided the car bodies permit free drainage.
(f) Provide additional stockpiling or binning in cases of high or non-uniform moisture.

1102.03 TEST METHODS.

Test aggregates covered by this subsection in accordance with the applicable provisions of subsection 1122.

1102.04 PREQUALIFICATION.

Aggregates covered by this subsection require a current Official Quality[5] meeting the requirements of this subsection as described in subsection 1101.04 prior to use on Department projects.

1102.05 BASIS OF ACCEPTANCE.

Aggregates covered by this subsection are accepted based on the procedure described in Subsection 1101.05

[5] See Section 1101.04 Pre-qualification, page 33 of this Appendix.

SUBSECTION 1103

FINE AGGREGATES FOR CONCRETE

1103.01 DESCRIPTION.

This specification covers fine aggregates for use in all types of concrete construction.

1103.02 REQUIREMENTS.

a. Composition.

(1) *Type FA-A.* Provide either singly or in combination a natural occurring sand resulting from the disintegration of siliceous or calcareous rock, or manufactured sand produced by crushing predominately siliceous materials.

(2) *Type FA-B.* Provide fine granular particles resulting from the crushing of zinc and lead ores (Chat).

b. Quality.

(1) Mortar strength. Compressive strength when combined with Type III (high early strength) cement.

(a) At age 24 hours, minimum.. 100%*

(b) At age 72 hours, minimum.. 100%*

* Compared to strengths of specimens of the same proportions, consistency, cement and standard 20-30 Ottawa sand.

(2) Hardening characteristics. Specimens made of a mixture of three parts *FA-B* and one part cement with sufficient water for molding will harden within 24 hours. There is no hardening requirement for *FA-A*.

c. Product Control

(1) Size Requirements. Provide aggregate that complies with the gradation requirements shown in Table 1103-1.

TABLE 1103-1. Gradation Requirements for Fine Aggregates for Concrete

Type	% Retained-Square Mesh Sieves						
	9.5 mm	4.75 mm	2.36 mm	1.18 mm	600 µm	300 µm	150 µm
FA-A	0	0-5	0-24	15-50	40-75	70-90	90-100
FA-B	0	0-5	0-24	15-50	40-75	70-90	90-100

(2) Deleterious Substances

(a) Type FA-A: Maximum allowed deleterious substances by mass are:

- Material passing the 75 µm sieve....................................2.0%
- Shale or Shale-like material, coal, soft or friable particles.........1.0%
- Sticks (wet)... 0.1%
- Clay lumps.. 0.25%

(b) Type FA-B: Provide materials that are free of organic impurities, sulfates, carbonates, or alkali. Maximum allowed deleterious substances by mass are:

- Material passing the 75 μm sieve....................................2.0%
- Clay or sludge lumps...0.25%

(3) Uniformity of Supply

Designate or determine the fineness modulus (gradation factor) in accordance with the procedure listed in the Construction Manual Part V, Section 17 prior to delivery or from the first 10 samples tested and accepted. Provide aggregate that is within ±0.20 of the average fineness modulus.

d. Proportioning of Coarse and Fine Aggregate

(1) Combine Fine and Coarse aggregates in a 50%-50% ratio by mass, with minor adjustments to improve workability when approved by the Engineer.

(2) Do not combine fine aggregate with coarse aggregate if either does not meet the requirements of 1104.02.b.(1). Consider such material, regardless of proportioning, as a Basic Aggregate and must conform to the requirements in Subsection 1104.

e. Handling Aggregates

(1) Stockpiling.

(a) Keep aggregates from different sources, with different gradings or with a significantly different specific gravity separated.

(b) Transport aggregate in a manner that ensures uniform gradation.

(c) Do not use aggregates that have become mixed with earth or foreign material.

(d) Stockpile or bin all washed aggregates produced or handled by hydraulic methods for 12 hours (minimum) before batching. Rail shipment exceeding 12 hours is acceptable for binning provided the car bodies permit free drainage.

(e) Provide additional stockpiling or binning in cases of high or non-uniform moisture.

1103.03 TEST METHODS.

Test aggregates covered by this subsection in accordance with the applicable provisions of subsection 1122.

1103.04 PREQUALIFICATION.

Aggregates covered by this subsection require a current Official Quality[6] meeting the requirements of this subsection as described in subsection 1101.04 prior to use on Department projects.

1103.05 BASIS OF ACCEPTANCE.

Aggregates covered by this subsection are accepted based on the procedure described in Subsection 1101.

[6] See Section 1101.04 Pre-qualification, page 33 of this Appendix.

SUBSECTION 1104

MIXED AGGREGATES FOR CONCRETE

1104.01 DESCRIPTION.

This specification covers mixed aggregates for use in all types of concrete construction. Mixed Aggregates are aggregates containing both coarse and fine material.

1104.03 REQUIREMENTS.

a. Composition.

(1) *Total Mixed Aggregate (TMA).* A natural occurring, predominately siliceous aggregate from a single source.

(2) *Mixed Aggregate*

(a) *Basic Aggregate (BA).* Singly or in combination, a natural occurring, predominately siliceous aggregate that does not meet the Wetting & Drying Test or gradation requirements of the Total Mixed Aggregate.

(b) *Coarse Aggregate Sweetener.* Types and proportions of aggregate sweeteners to be used with *BA* are listed in Table 1104-1.

Table 1104-1

Type of Coarse Aggregate Sweetener	Proportion required Minimum % by mass
Crushed Sandstone	30
Chat	25
Crushed Limestone or Dolomite	30
Total Mixed Aggregate	30

Waive the minimum portion of coarse aggregate sweetener for all *BA* that meet the wetting and drying requirements for *TMA*. In this case, combine the *BA* and coarse aggregate sweetener in proportions required to meet the gradations listed in 1104.02.c(1).

b. Quality.

(1) Total Mixed Aggregate.

(a) Soundness, minimum..0.90

(b) Wear, maximum...50%

(c) Wetting & Drying Test for Total Mixed Aggregate

Concrete Modulus of Rupture:

At 60 days, minimum... 3.8 MPa

At 365 days, minimum .... 3.8 MPa

Expansion:

At 180 days, maximum 0.050%

At 365 days, maximum.. 0.070%

(2) Basic Aggregate.

(a) Retain 10% or more of the *BA* on the 2.36 mm sieve before adding the Coarse Aggregate Sweetener. Aggregates with less than 10% retained on the 2.36 mm sieve are to be considered a Fine Aggregate described in Subsection 1104. Furnish material with less than 5% calcareous material retained on the 9.5 mm sieve.

(b) Soundness, minimum..0.90

(c) Wear, maximum 50%

(d) Mortar Strength. Compressive strength on material passing the 9.5 mm sieve when combined with Type III (high early strength) cement.

At age 24 hours, minimum...100%*

At age 72 hours, minimum...100%*

* Compared to strengths of specimens of the same proportions, consistency, cement and standard 20-30 Ottawa sand.

(3) Coarse Aggregate Sweetener

(a) Must meet the requirements for CA-6 described in Subsection 1102.

c. Product Control

(1) Size Requirements. The grading must comply with the requirements shown in Table 1104-2.

TABLE 1104-2 Grading Requirements for Mixed Aggregates for Concrete

Type	Usage	% Retained-Square Mesh Sieves									
		25.0 mm	19.0 mm	12.5 mm	9.5 mm	4.75 mm	2.36 mm	1.18 mm	600 µm	300 µm	150 µm
MA-1	All concrete except mainline pavement [1]	0	0-5	-	-	20-60	-	-	76-84	90-96	-
MA-2	All Concrete	-	0	5-10	19-29	36-46	53-63	67-77	80-88	89-97	98-100

1 MA-1 can be used in concrete for mainline patching.

(2) Deleterious Substances

Maximum allowed deleterious substances by mass are:

- Material passing the 75 µm sieve.. 2.5%
- Shale or Shale-like material...0.5%
- Soft or friable particles.. 2.5%
- Sticks (wet)...0.1%
- Coal... 0.5%
- Clay lumps 0.5%

None of the individual components listed can be exceeded. Combinations excluding the 75 µm requirement cannot exceed 3.0%.

(3) Uniformity of Supply
Designate or determine the fineness modulus (gradation factor) in accordance with the procedure listed in the Construction Manual Part V, Section 17 prior to delivery or from the first 10 samples tested and accepted. Provide aggregate that is within ±0.20 of the average fineness modulus.

d. Handling Aggregates
(1) Segregation.
Remix all aggregate that has segregated through transit or stockpiling before acceptance testing.
(2) Stockpiling.
(a) Keep aggregates from different sources, with different gradings or with a significantly different specific gravity separated.
(b) Transport aggregate in a manner that ensures uniform gradation.
(c) Do not use aggregates that have become mixed with earth or foreign material.
(d) Stockpile or bin all washed aggregates produced or handled by hydraulic methods for 12 hours (minimum) before batching. Rail shipment exceeding 12 hours is acceptable for binning provided the car bodies permit free drainage.
(e) Provide additional stockpiling or binning in cases of high or non-uniform moisture.

1104.03 TEST METHODS.

Test aggregates covered by this subsection in accordance with the applicable provisions of subsection 1117.

1104.04 PREQUALIFICATION.

Aggregates covered by this subsection require a current Official Quality[7] meeting the requirements of this subsection as described in subsection 1101.04 prior to use on Department projects.

1104.05 BASIS OF ACCEPTANCE.

Aggregates covered by this subsection are accepted based on the procedure described in Subsection 1101.05.

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[7] See Section 1101.04 Pre-qualification, page 33 of this Appendix.

APPENDIX C

KTMR-23 WETTING AND DRYING TEST OF SAND AND SAND-GRAVEL AGGREGATE FOR CONCRETE

a. SCOPE

This test shall be used to determine the acceptability of sand and sand-gravel aggregate to be used in concrete construction, both pavement and structural.

b. REFERENCED DOCUMENTS

b.1. AASHTO T 119; Slump of Hydraulic Cement Concrete

b.2. AASHTO T 126; Making and Curing Concrete Test Specimens in the Laboratory

b.3. AASHTO T 140; Compressive Strength of Concrete Using Portions of Beams Broken in Flexure

b.4. AASHTO T 177; Flexural Strength of Concrete [Using Simple Beam With Center Point Loading]

b.5. AASHTO M 231; Balances Used in the Testing of Materials

c. APPARATUS

c.1. Molds suitable for casting 76.2 X 101.6 X 406.4 mm (3 X 4 X 16 in) beams.

c.2. Rotary concrete mixer as specified in AASHTO T 126.

c.3. A balance of sufficient capacity conforming to requirements of AASHTO M 231.

c.4. Slump cone and rod as specified in AASHTO T 119.

c.5. A drying oven capable of maintaining a temperature of 53.3-54.4°C (128-130°F).

c.6. Water bath capable of maintaining a temperature between 15.6-26.7°C (60-80°F).

c.7. Length comparator capable of accurately reading beams to the nearest 0.01 mm (0.001 in).

c.8. A testing machine for modulus of rupture determination as specified in AASHTO T 177.

c.9. A 15.9 mm (5/8 in) diameter steel rod having a hemispherical tip the same diameter as the rod.

d. SAMPLE PREPARATION

d.1. Cement:[a] Use Monarch, Type II cement. If not available, then use the cement type and brand designated by the Engineer of Tests.

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[a] The requirement for Monarch Type II cement exists because of its alkali level is as close to, but not exceeding, the 0.6% maximum

d.2. The gradation of the aggregate shall be within the middle 1/3 of the limits specified for MA-1 (**Table 1**) except for the 19 mm (3/4 in) sieve. It shall be further prepared by screening over the 19 mm (3/4 in) sieve and all material retained on the 19 mm (3/4 in) sieve shall be crushed to pass the 19 mm (3/4 in) sieve and incorporated into the mix.

Table KT-MR-1. Gradation Requirements for Wetting and Drying Test

MA-1 Percent Retained - Square Mesh Sieves								
19.0 mm (¾ in.)	12.5 mm (½ in.)	9.5 mm (3/8 in.)	4.75 mm (No. 4)	2.36 mm (No. 8)	1.18 mm (No. 16)	600 µm (No. 30)	300 µm (No. 50)	150 µm (No.100)
0-5	...	...	20-60	...	...	76-84	90-96	...

d.3. Run the specific gravity and absorption tests in accordance with KT-6-94 procedure I & II of the Part V Construction Manual. Run tests on the as-received material.

d.3.a. Using the results from the specific gravity and absorption tests, determine the average specific gravity and absorption in a 40 / 60 mix of dry material. The mix represents 40% being + 4.75 mm (+ 4) material and 60% being - 4.75 mm (- 4) through + 75µm (+ 200) material.

d.3.b. Recombine the material to the following schedule to produce three 18.145 kg (40 lb) batches.

12.5 mm (1/2") - 0.363 kg (0.8 lb)
9.5 mm (3/8") - 2.359 kg (5.2 lb)
4.75 mm (#4) - 4.536 kg (10.0 lb)
2.36 mm (#8) - 2.722 kg (6.0 lb)
1.18 mm (#16) - 1.814 kg (4.0 lb)
600 µm (#30) - 2.722 kg (6.0 lb)
300 µm (#50) - 2.359 kg (5.2 lb)
150 µm (#100) - 0.907 kg (2.0 lb)
75 µm (#200) - 0.363 kg (0.8 lb)

Total -18.145 kg (40.0 lb)

d.3.c. Place the material into galvanized or rust resistant pans, add the amount of water equal to the absorption and mix uniformly. Cover the material with a plastic sheet and let stand for approximately 4 hours in order to reach a saturated surface dry condition.

d.4. Create a concrete mix having a water/cement ratio of 0.51 and having a slump of 50.8 mm (2 in) and 76.2 mm (3 in). Place two 18.145 kg (40 lb) batches of aggregate, design weight of cement and water in the mixer and start mixing. Using the third aggregate batch to bring mix to the desired slump.

d.5. Cast six 76.2 X 101.6 X 406.4 mm (3 X 4 X 16 in) beams as described below, and remove from the molds within 24 ± 8 hours from time of casting. Beams should be protected from loss of moisture during mold removal. Identify each beam for future tracking.

d.5.a. Place the concrete in the molds taking care to ensure each scoop is representative of the mix. Move the scoop around the edge of the mold as the concrete is discharged to minimize segregation and to ensure uniformity of distribution. Further distribute the concrete by use of a tamping rod prior to consolidation. Do not add non-representative concrete to an underfilled mold.

d.5.b. Place the concrete in the mold in two layers of approximately equal volume. Rod each layer 32 times with the rounded end of the rod. Rod the bottom layer throughout its depth, distributing the strokes uniformly over the cross section of the mold. For the upper layer, allow the rod to penetrate about 12.7 mm (1/2 in) into the bottom layer. After each layer is rodded, spade the concrete around the edges of the mold with a trowel or spatula. The molds containing the concrete shall then be tapped lightly on the table top to close any remaining voids. Finish the surface with a wood float using the minimum amount of manipulation necessary to produce a plane surface that is essentially level with the top edge of the mold.

d.6. Cure the beams seven days in a moist room maintained at 23 ± 2.2°C (73.4 ± 3°F) and at not less than 95% relative humidity, then 21 days in air at a temperature between 20-27.5°C (68-81.5°F) and 50% relative humidity.

d.7. At 28 days obtain cured (dry) mass and length. Place beams in water bath maintained at 15.6-26.7°C (60-80°F) for a minimum of 1 hour. Obtain mass in water and saturated surface dry (SSD) to determine the specific gravity as specified in **g.1.** Place beams back in water bath for 48 hours.

NOTE b: Differences in specific gravity between the six beams can be an indication of air entrapment or poor consolidation in specimens.

d.7.a. During the length determination, select the three best fitting beams for 365-day cycling. Best fitting pertains to the ability of the beam to fit in the comparator with pins fully aligned and minimal rocking motion.

d.8. The beams to be tested in flexure at 60 days shall then be cured in the moist room for an additional 30 days.

e. PROCEDURE

e.1. Measure length of beams at the following ages: 30, 60, 120, 180, 240, 300, and 365 days. Make every attempt to choose a time when the 30, 60 and 365 day checks can be guaranteed. Other dates should fall within plus or minus one day. At each age the beams shall be submerged in water maintained between 15.6-26.7°C (60-80°F) for not less than 15.5 ± 0.5 hours prior to measurement.

e.2. Sixty days after casting, test the three beams cured in the moist room for modulus of rupture as specified in AASHTO T 177. Conduct the test with the 76.2 X 406.4 mm (3 X 16 in) faces perpendicular to the applied load, with the load applied at the center of a 355.6 mm (14 in) span.

e.2.a. Upon completing the modulus of rupture test, break both halves of the beams in accordance to AASHTO T 140 (See **Figure 1**).

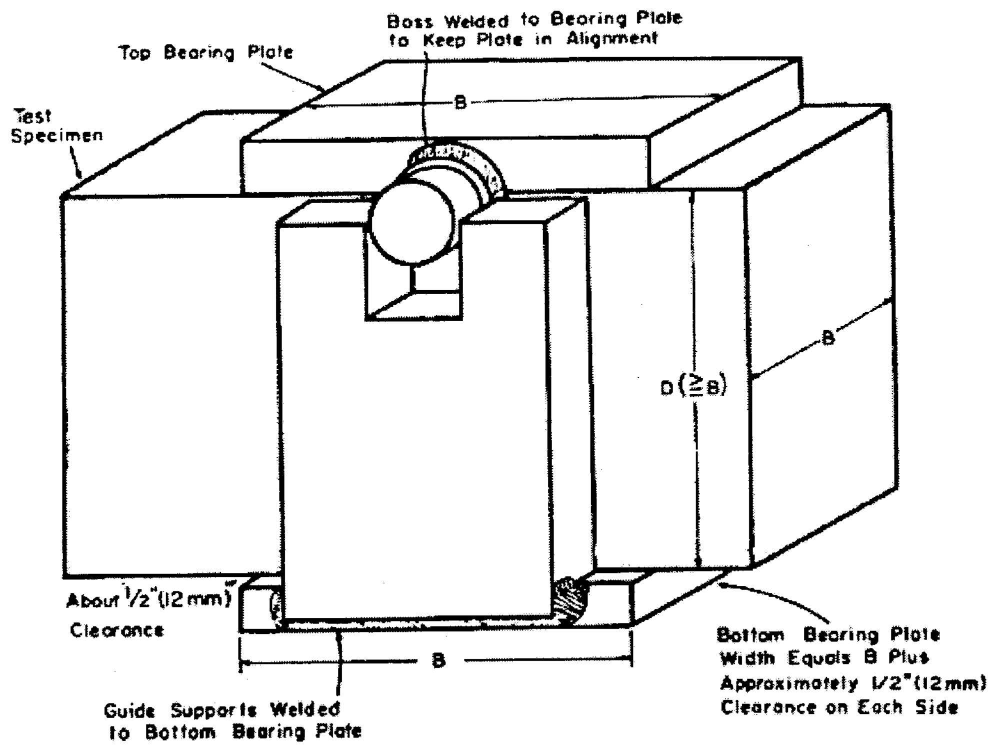

Figure 1

e.3. Beginning 30 days after casting, subject the other three beams to the following wetting and drying test procedure.

e.3.a. Place the beams in the oven maintained at 53-54°C (128-130°F) for eight hours.

e.3.b. Remove the beams from the oven and submerge them in the water bath at 16-27°C (60-80°F) for 15.5 ± 0.5 hours. Procedure (**e.3.a.**) and (**e.3.b.**) constitutes one cycle and shall be completed in 24 hours.

e.3.c. Repeat the cycle each consecutive day throughout the 365-day period except for weekends and holidays when the beams are to remain in the water bath.

e.4. Calculate and record the length change, expressed as percent expansion, at each of the ages stated under (**e.1.**) using the length measured at 30 days as the base as specified in **g.2.**

e.5. The beams shall be tested for modulus of rupture, upon completion of the 365-day test. The test shall be conducted with the 76.2 X 406.4 mm (3 X 16 in) faces perpendicular to the applied load, with the load applied at the center of a 355.6 mm (14 in) span as specified in AASHTO T 177.

e.6. Upon completing the modulus of rupture test, subject each half to a compressive strength test in accordance to AASHTO T 140.

f. REQUIREMENTS FOR ACCEPTABILITY OF THE AGGREGATE

f.1. Each of the two groups of beams tested in flexure at 60 days and 365 days shall have an average modulus of rupture of not less than 3.8 MPa (550 psi).

f.2. Expansion of beams:

f.2.a. At 180 days, the increase in length shall not exceed 0.050%.

f.2.b. At 365 days, the increase in length shall not exceed 0.070%.

g. CALCULATIONS

g.1. Bulk Specific Gravity:

$$G_{sb} = \frac{A}{B - C}$$

Where:
A = Mass of cured beam, g
B = Saturated surface-dry beam, g
C = Mass of beam in water, g

g.2. Percent expansion of beam:

$$\Delta L\% = \frac{100(L_n - L_{30})}{L_{30}}$$

Where:
$\Delta L\%$ = Percent change in length
L_{30} = Length of specimen at 30 days
L_n = Length of specimen at n days (n=60, 120, 180, 240, 300, or 365 days)

h. REPORT

See attached report.

KANSAS DEPARTMENT OF TRANSPORTATION

Sample of Sand Gravel (Wetting Drying)

Laboratory No 96-4776
CMS No.
Date reported
Date received 10/21/96

Spec. No. 1990 SS, Subsec. 1102 (b) (1.1.3) Qty Unlimited
Property of
Sample from
Submitted by J. Frantzen Topeka, KS
Ident. Marks

Project No. Wetting & Drying Co/Dt Type

Contractor

TEST RESULTS

This material was tested in accordance with Article 1117 (t) of the 1990 KDOT Standard Specifications using Type II cement.

MATERIALS:

Aggregate - MA-1

-

Cement - Monarch type I/II, Lab. #XX-XXXX

AGGREGATE SIEVE ANALYSIS:

SIEVE ANALYSIS OF MA - 1										
Metric	mm						µm			
	19.0	12.5	9.5	4.75	2.36	1.18	600	300	150	75
English	in.									
	(3/4)	(1/2)	(3/8)	(#4)	(#8)	(#16)	(#30)	(#50)	(#100)	(#200)
% Ret.	0	2	15	40	55	65	80	93	98	100

Agg. Specific Gravity, S.S.D. (Theo. Comb.) ---------- 2.58
% Absorption (Theo. Comb.) ---------- 1.46
-#200 Material (%) ---------- 0.00

Laboratory No 96-4776

MIX DESIGN DATA:

Date Made		11/18/96
Cement, kg (lb)	42.64	(94.00)
Water, kg (lb)	21.74	(47.94)
MA-1, kg (lb)	253.14	(558.09)
Slump, mm (in)	57.2	(2.25)

Time of slump after addition of water (min.) ---- 12:15

Unit Weight:

Theoretical Air Free, kg/m^3 (lb/ft^3)	2380.8	(148.63)
Actual, kg/m^3 (lb/ft^3)	2325.2	(145.16)

Air Content:

Gravimetric, %	2.3
Rollameter, %	3.5

Yield Cement Factor kg (lb)	238.72	(526.29)
Water - Cement Ratio, kgs/kg (lbs/lb)		0.51

TEST DATA:

Note: The corrected modulus of rupture MPa (psi) is for information only.

Specimen	Mod. of Rupture MPa (PSI) Uncorrected	Corrected	Change in Length (%)	Fund, Frequency (%30 day reading)
A	5.37 (779)	4.90 (710)		
B	5.34 (775)	4.96 (720)		
D	5.57 (808)	5.47 (794)		
Avg. @ 60 days	5.43 (787)	5.11 (741)		
C			0.027	
D			0.027	
E			0.027	
Avg. @ 179 days			0.027	109..31
C	5.10 (740)	5.10 (740)	0.053	
E	4.81 (698)	4.73 (686)	0.040	
F	3.76 (546)	3.63 (526)	0.047	
Avg. @ 365 days	4.56 (661)	4.49 (651)	0.047	108.81

Laboratory No 96-4776

COMPRESSIVE STRENGTH:

		Unit Load		Avg. Unit Load	
Specimen	Age (days)	MPa	(PSI)	MPa	(PSI)
A	60	43.02	(6240)		
A	60	44.54	(6460)		
B	60	42.82	(6210)		
B	60	41.37	(6000)		
D	60	39.85	(5780)		
D	60	43.64	(6330)	42.54	(6170)
C	365	43.44	(6300)		
C	365	42.82	(6210)		
E	365	41.58	(6030)		
E	365	38.75	(5620)		
F	365	40.82	(5920)		
F	365	42.47	(6160)	41.64	(6040)

NOTE: 232 cycles of wetting & drying.

DISPOSITION:

This material meets the requirements of Article 1117(t) of the 1990 KDOT Standard Specifications and is approved for use under the requirements of Sub-Article 1102 (b) (1.1.3) .

cc: L. S. Ingram (3)
File (2)

Reported by: ______________________.

L. C. Schroeder, PhD., P.E.
Title: Engineer of Physical Tests.